CITY OF NEWPORT

ORDINANCE NO. 2049

AN ORDINANCE REPEALING AND REPLACING THE PUBLIC FACILITIES AND URBANIZATION ELEMENTS OF THE NEWPORT COMPREHENSIVE PLAN ORIGINALLY ADOPTED BY ORDINANCE NO. 1621
(Newport File No. 3-CP-12)

Summary of Findings:

1. On December 10, 2012 the Newport Planning Commission initiated amendments to the "Public Facilities" and "Urbanization" elements of the Newport Comprehensive Plan to update standards against which an Urban Growth Boundary (UGB) amendment is evaluated; establish policies to acquire lands and protect water quality within the city's municipal watershed; acknowledge structural deficiencies in the City's municipal water reservoirs; and outline steps the City will take to resolve the structural deficiencies.

2. Newport City Council desires to expand the UGB to include Big Creek Reservoir #1 and Big Creek Reservoir #2, which are the City's primary storage facilities for its domestic water supply. This expansion is desirable because placing the land under a "Public" Comprehensive Plan and zoning designation will make it easier for the City to modify its water infrastructure in response to known structural deficiencies at the reservoirs and to construct a future regional park as envisioned in the 1993 Park System Master Plan.

3. Repealing and replacing the "Public Facilities" element of the Newport Comprehensive Plan sets the table for an expansion proposal. Preliminary geotechnical analysis, prepared by HDR Consultants, describes the nature of structural deficiencies inherent to Big Creek Reservoir #1 and Big Creek Reservoir #2, and supports the adoption of policies describing how the City should respond to this threat to its domestic water supply. Proposed policies provide direction for completing necessary engineering studies to ascertain the full scope of the problem, financing future construction and land acquisition, and protecting water quality consistent with a source water assessment performed by the Oregon Department of Environmental Quality and Oregon Health Department.

4. Similarly repealing and replacing the "Urbanization" element of the Newport Comprehensive Plan sets the table for an expansion proposal and is necessary because it updates outdated criteria for evaluating such requests to that the standards conform to current state law, namely Statewide Planning Goal 14, amended April of 2006.

5. The Newport Comprehensive Plan element entitled "Administration of the Plan" lists factors that must be met to amend the document, such factors being listed explicitly in the Planning Staff Memorandum dated, January 23, 2013 and incorporated herein.

a. The revised "Public Facilities" element satisfies the listed factors in that it updates technical inventories related to the structural integrity of Big Creek Reservoir #1 and Big Creek Reservoir #2 and the quality of the water within the municipal watershed, and puts in place policies and implementation strategies that respond to the new information.

b. The revised "Urbanization" element satisfies the listed factors in that it updates the City's criteria for evaluating UGB amendment proposals to be consistent with current state law.

6. Repealing and replacing the "Public Facilities" and "Urbanization" elements of the Newport Comprehensive Plan are consistent with applicable Statewide Planning Goals in that the changes:

a. Have been developed and vetted with the City of Newport Planning Commission and its Advisory Committee consistent with Statewide Planning Goal 1, Public Involvement; and

b. Update the Newport Comprehensive Plan's technical inventory (with respect to the condition of the reservoirs and water quality) and criteria (with respect to UGB amendments) that facilitate a land use planning process and policy framework that provides an adequate factual basis for decision making consistent with Statewide Planning Goal 2, Land Use Planning; and

c. Ensure that the Newport Comprehensive Plan contains accurate information about the condition of the City's water infrastructure as encouraged by Statewide Planning Goal 11, Public Facilities and Services; and

d. Put in place standards for amending the Newport Urban Growth Boundary consistent with ORS 197.298 and the following factors (1) efficient accommodation of identified land needs; (2) orderly and economic provision of public facilities and services; (3) comparative environmental, energy, economic and social consequences; and (4) compatibility of the proposed urban uses with nearby agricultural and forest activities occurring on farm and forest land outside the UGB, as set out in Statewide Planning Goal 14, Urbanization.

7. No other Statewide Planning Goals are applicable to the proposed changes to the "Public Facilities" and "Urbanization" sections of the Newport Comprehensive Plan.

8. The Newport Planning Commission reviewed the proposed changes to the "Public Facilities" and "Urbanization" sections of the Newport Comprehensive Plan, as they were

being developed, at work sessions on October 8, 2012, October 22, 2012, November 26, 2012, and December 10, 2012. The Planning Commission held a public hearing on January 28, 2013, and voted to recommend adoption of the amendments.

9. The City Council held a public hearing on February 19, 2013 regarding the question of the proposed revisions, and voted in favor of their adoption after considering the recommendation of the Planning Commission and evidence and argument in the record.

10. Information in the record, including affidavits of mailing and publication, demonstrate that appropriate public notification was provided for both the Planning Commission and City Council public hearings.

THE CITY OF NEWPORT ORDAINS AS FOLLOWS:

Section 1. The Public Facilities element of the Newport Comprehensive Plan, originally adopted by Ordinance No. 1621 (as amended) is repealed and replaced with the text at Exhibit A, attached to this Ordinance.

Section 2. The Urbanization element of the Newport Comprehensive Plan, originally adopted by Ordinance No. 1621 (as amended) is repealed and replaced with the text at Exhibit B, attached to this Ordinance.

Section 3. The document titled "Big Creek Dam No. 1 and No. 2, Preliminary Geotechnical Investigation and Seismic Evaluation" and prepared by HDR Consultants in February of 2013, attached as Exhibit C, is included as support for this ordinance.

Section 4. The Planning Staff Memorandum dated January 23, 2013, attached as Exhibit D, is included as support for this ordinance.

Section 5. This Ordinance shall take effect 30 days after passage.

Date adopted and read by title only: February 19, 2013

Signed by the Mayor on February 20, 2013.

Sandra Roumagoux

Sandra Roumagoux, Mayor

ATTEST:

Margaret M. Hawker

Margaret M. Hawker, City Recorder

EXHIBIT A
Ordinance No. 2049
(File No. 3-CP-12)

GOALS AND POLICIES
PUBLIC FACILITIES ELEMENT

GENERAL

Goal: To assure adequate planning for public facilities to meet the changing needs of the City of Newport urbanizable area.

Policy 1: The city shall develop and maintain public facilities master plans (by reference incorporated herein). These facility plans should include generalized descriptions of existing facilities operation and maintenance needs, future facilities needed to serve the urbanizable area, and rough estimates of projected costs, timing, and probable funding mechanisms. Public facilities should be designed and developed consistent with the various master plans.

Policy 2: In order to assure the orderly and cost efficient extension of public facilities, the city shall use the public facilities master plans in the capital improvement planning.

Policy 3: The city shall work with other providers of public facilities to facilitate coordinated development.

Policy 4: Essential public services should be available to a site or can be provided to a site with sufficient capacity to serve the property before it can receive development approval from the city. For purposes of this policy, essential services shall mean:

> Sanitary Sewers

> Water

> Storm Drainage

> Streets

Development may be permitted for parcels without the essential services if:

> The proposed development is consistent with the Comprehensive Plan; and

> The property owner enters into an agreement, that runs with the land and is therefore binding upon future owners, that the property will connect to the essential service when it is reasonably available; and

> The property owner signs an irrevocable consent to annex if outside the city limits and/or agrees to participate in a local improvement district for the essential service.

Policy 5: Upon the annexation of territory to the City of Newport, the city will be the provider of water and sewer service except as specified to the contrary in an urban service agreement or other intergovernmental agreement.

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WATER

Goal: To provide the City of Newport with a high quality water system that will supply residents and businesses with adequate quantities for consumption and fire protection.

Policy 1: The city will comply with state and federal laws concerning water quality and will take appropriate steps consistent with those laws to protect and maintain drinking water source areas.

Implementation Measure 1: The City shall work to establish a source water protection buffer in the Big Creek Watershed. The City declares the Big Creek Watershed a public facility consistent with the definition of Public Facility Systems in OAR 660-011-0005(7)(a)(A). The City will work to establish a source water protection buffer that is consistent with the findings of the Oregon Department of Environmental Quality / Oregon Health Department source water assessment report (PWS #4100566).

Policy 2: The water system will be designed and developed to satisfy the water demand of the various users under normal and predictable daily and seasonal patterns of use, and at the same time provide sufficient supplies for most emergency situations.

Policy 3: The city may extend water service to any property within the city's urban growth boundary, and may extend water service beyond the urban growth boundary if the extension of service is not inconsistent with an urban service agreement or other intergovernmental agreement. The city may require a consent to annexation as a condition of providing water service outside the city limits.

Policy 4: The city will acquire lands within the municipal watershed when available or necessary to protect water quality or improve its water system.

Policy 5: The city will reconstruct its municipal raw water storage and distribution facilities to address identified structural deficiencies to Big Creek Dam #1 and Big Creek Dam #2.

Implementation Measure 1: The city shall conduct necessary and appropriate engineering studies to determine the safest and most cost-effective approach to ensure the integrity of the municipal water supply. The studies shall identify the cost and timing of needed capital projects to address identified structural deficiencies and comply with Policy 2 of this section.

Implementation Measure 2: The city shall explore financing mechanisms, and prepare a financing plan to fund construction needed to resolve the structural deficiencies by 2030.

Implementation Measure 3: The city shall use data and findings from Implementation Measures 1 and 2 of this section to update the Water Supply section of the Public Facilities element of the Newport Comprehensive Plan to reflect new information as a result of the engineering and finance studies.

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WASTEWATER

Goal: To provide a wastewater collection and treatment system with sufficient capacity to meet the present and future needs of the Newport urbanizable area in compliance with State and Federal regulations.

Policy 1: On-site sewer systems shall not be allowed unless the city's sanitary sewer system is greater than 250 feet away. In any case, a subsurface permit from
the Lincoln County Sanitarian must be obtained prior to any development that will rely on an on-site sewer system.

Policy 2: City wastewater services may be extended to any property within the urban growth boundary. Except for the very limited circumstances allowed by state law and regulations, the city will not generally provide wastewater services outside the urban growth boundary. The city may require a consent to annexation as a condition of providing wastewater service outside the city limits. Nothing in this policy obligates the City to provide wastewater services outside of the city limits. For property outside the city limits but within the urban growth boundary, wastewater services may be provided at the City's discretion only for:

a) residentially zoned lands as allowed by county zoning without full services, and

b) commercial and industrial zoned lands to existing lawful uses as of the date (9/4/07) of this amendment.

Policy 3: The city will design and develop the wastewater collection and treatment system in a way that addresses the demands of the various users under normal and predictable daily and seasonal patterns of use.

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TRANSPORTATION

Transportation Goals and Policies repealed by Ordinance No. 1802 (January 4, 1999).

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STORM WATER DRAINAGE

Goal: To provide a storm water drainage system with sufficient capacity to meet the present and future needs of the Newport urbanizable area.

Policy 1: The city will comply with state and federal laws concerning water quality.

Policy 2: The city will use existing, natural drainage systems to the greatest extent possible.

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AIRPORT

Goal: To provide for the aviation needs of the City of Newport and Lincoln County.

Policy 1: The city will ensure through zoning and subdivision ordinance provisions that the airport will be able to operate safely and efficiently.

Policy 2: The city will cooperate with state and federal agencies in the development of the airport.

URBANIZATION

The Newport urban area includes lands within the city limits. It becomes necessary, however, to identify lands outside those limits that will become available for future growth. With that in mind, the City of Newport and Lincoln County have agreed upon a site specific boundary that limits city growth until the year 2031.

The urban growth boundary (UGB) delineates where annexations and the extension of city services will occur. Converting those county lands within the UGB requires coordination between the county, the property owners, and the city. This section provides the framework and the policies for those conversions and service extensions. The decision makers can also use this section as a guide for implementation of the urbanizing process.

The city and county made the policies of this section as part of a coordinated effort. Involved in the process were the governing bodies and planning commissions of both jurisdictions. The Citizen's Advisory Committee, concerned citizens, and other affected agencies also participated in the process.

Newport Urban Growth Areas:

Land forms are the most important single determinant of the directions in which Newport can grow. Newport is bounded on the west by the Pacific Ocean and on the east by the foothills of the Coast Range. In addition, the city is divided by Yaquina Bay. The only suitable topography for utility service and lower cost urban development is along the narrow coastal plain. Some development has occurred in the surrounding foothills and along the Yaquina River and creek valleys, but this is generally rural development of low density without urban utilities. The following inventory describes areas evaluated as to their suitability to accommodate expected growth.

A. Agate Beach Area (North Newport/390 Acres):

Inventory. This study area consists of both urbanized and undeveloped land (see map on page 283). Of the 390 acres available for residential development, 225 lie within the unincorporated area of the UGB, and 165 acres are within Newport's city limits. (The urbanized area contains approximately 60 acres.)

The urbanized area was platted in the 1930's, with growth occurring gradually since that time. The area is primarily residential and has a mixture of houses, mobile homes, trailers, and some limited commercial uses along U.S. Highway 101. The area was previously served by the Agate Beach Water System, which frequently failed to meet federal water quality standards and had inadequate line size and pressure to serve existing customers and projected growth. The City of Newport rebuilt the water system and installed a sewer system at the cost of approximately $1.4 million.

The unincorporated portions of this study area have been included in Newport's UGB to help meet anticipated need for residential land. The land is relatively level, water services

and road access are immediately adjacent, and sewer is available. The area has been urbanized to a degree already and is suitable for continued residential development. Much of this area has been platted into 5,000 square foot lots, which are both suitable for mobile home placement and "buildable" as sewer is extended.

Analysis. Because most of this area has been previously platted into 50 x 100 foot lots, land costs can be expected to be lower than in newly platted areas of the city. Many mobile homes and trailers currently exist in this area, and smaller lots are appropriate for mobile homes.

Finding. This area is suitable for continued residential development and is designated residential. In addition, because of the smaller lot sizes and the existence of many mobile homes in the area, a mobile home overlay zone is desirable and compatible with existing uses. Areas of larger acreage on both the east and west side are suitable for high density residential use with the mobile home overlay so that new mobile home parks may be built in the area as outright uses, as well as allowing apartments. Existing commercial development along U.S. Highway 101 should be allowed to remain.

B. Agate Beach Golf Course and Little Creek Drainage Area (North Newport/93 acres):

Inventory. This area lies south and east of the golf course, west of the west line of Section 33, and east of Highway 101, all of which is within the city limits (see map on page 283). The area is generally undeveloped, and it slopes steeply toward Little Creek.

The area has been planned to be served by city water and sewer and a major new road. It is zoned for low and high density residential development.

Analysis. Because of the steep slopes, this is the type of area where a planned development is often appropriate. It borders a mobile home park to the south and is geographically well separated from other areas of conventional housing; therefore, mixed residential development can be considered for the property with little possible conflict.

Finding. Because of the topography, either low density residential development with a planned development overlay or high density residential development would be appropriate designations. However, the former would insure more open space in the long range.

C. West Big Creek Drainage Area (North Newport/40 acres):

Inventory. This area lies south of the Pacific Beach Club, east of U.S. Highway 101, and west of Lakewood Hills (see map on page 283). It has not yet been developed.

Analysis. Much of the area is in a flood plain. However, it has been studied for a planned development and is suitable for high density residential use.

Finding. High density residential will be the designation for this property. The land may be suitable for a planned unit development.

D. East Big Creek Drainage Area (City Reservoir):

Inventory. This area drains into the city reservoir, and the city owns the majority of the land (see map on page 283). There are several smaller private parcels with houses and livestock.

Finding. This area could eventually be used as a large city park or residential area once the reservoir is no longer used for the city water supply. During the planning period, this area should be protected from further residential development.

That land which is not needed for public park land shall be considered for return to the private sector for housing.

E. Jeffries Creek Drainage Area (Northeast Newport/220 Acres):

Inventory. This area is south of the city reservoir, north of Old Highway 20, east of Harney Street, and west of the eastern half of Section 4 (see map on page 283). This area contains the Terrace Heights, Virginia Additions, Kewanee Addition, and the Beaver State Land property. There is very little development in the area as yet. Fifty-five acres lie within Newport's city limits.

Analysis. Platted around the turn of the century, this area has long been planned for low density residential development. Little has occurred so far due to more accessible development closer to Newport. This is no longer the case, and this land is now needed for housing.

Finding. This area has steep slopes, no existing utilities as yet, and will be expensive to develop. However, much of the property will have ocean or bay view. The area is appropriate for low density development.

F. Harbor Heights Area (Southeast Newport/267 Acres):

Inventory. This study area lies east of Harbor Heights to the urban growth boundary and north of Bay Road to the urban growth boundary (see map on page 283). Of its 267 acres, approximately 44 are within Newport's city limits.

Analysis. This is an area where lot sizes might well be raised to a higher minimum to encourage the maintenance of the vegetation that helps stabilize the entire area. This would be a high cost housing area with very low density development.

Finding. The area is steep with some slide potential. Dotted with residential uses, the area commands a view of the bay and is in heavy demand. A low density residential designation is appropriate for this area.

G. Idaho Point Area (South Beach/120 Acres):

Inventory. This area stretches from South Bay Street to the Idaho Point Marina and from S.E. 32nd Street south to the forest lands (see map on page 283).

Analysis. The existing water system is inadequate and is being replaced, along with city sewer. Some of the area is in demand for its bay view, and much of the land could be developed for medium to high cost housing. The topography varies from flat to steeply sloping, with most in the in between category; therefore, development costs will vary.

Finding. The topography in the area varies from flat to steeply sloping, with most of it moderately sloping. The existing water system is inadequate and sewer is not yet available. Some low density residential uses currently exist, and the area has been planned for a mix of low and high density residential.

H. South Beach (South of Newport/560 Acres):

Inventory. The area extends from S.E. 32nd Street to the southern boundary of the Newport Municipal Airport and from the southerly extension of Bay Street to U.S. Highway 101 (see map on page 283).

Analysis. The area has long been planned for urban development and is currently coming along in that manner. Newport has planned for many years to encourage industrial development in South Beach.

Finding. It is the only area for which the city has planned industrial development that would allow non-water related or non-water dependent industrial development. The area will need city sewer and other city services.

I. Wolf Tree Destination Resort (South of Newport/1,000 Acres):

Inventory. The city extended its urban growth boundary and the city limits to include about 1,000 acres for the Wolf Tree Destination Resort consistent with Goal 8 (see map on page 284). The area includes about 800 acres south of the Newport Municipal Airport, with another 200 acres lying east of the airport. The region has a special plan and zoning designation that limits the land for a destination resort.

Analysis. Currently undeveloped except for a few scattered residences, the area has been planned for a destination resort since 1987. The south area is presently in the city limits, but the easterly 200 acres is not. The Wolf Tree property was brought into the UGB and annexed to the city only after a Goal 8 Destination Resort analysis and a limitation on

the property to the development of a destination resort. Many state and federal agencies were involved in the process that brought this property into the UGB and the city limits.

Finding. The project complies with Goal 8/"Destination Resort." The property cannot be developed except as a destination resort consistent with state and city law.

Finding. The City of Newport has established its urban growth boundary as indicated on the city's Comprehensive Plan Map (available in the city's Planning Department office), in accordance with the following findings and as demonstrated in the inventory:

> The projected population growth requirements of the City of Newport, as demonstrated in the inventory, cannot be met within the existing city limits.

> In order to provide adequate housing opportunities and needed employment and to plan for a livable environment, there is a need for additional acreage beyond that currently available within the Newport city limits.

> The City of Newport has planned for the urbanization of the UGB area based upon the city's long-range plan and capacity to extend needed facilities and service during the planning period.

> In determining the most appropriate and efficient land uses and densities within the UGB, the City of Newport has considered current development pattern limitations posed by land forms, as well as the city's needs during the planning period.

> In establishing its UGB, the City of Newport has considered and accounted for environmental, energy, economic, and social consequences as demonstrated in the inventory.

> There are no agricultural lands adjacent to the Newport urban growth boundary.

> What alternative locations within the area have been considered for the proposed needs.

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GOALS/POLICIES/IMPLEMENTATION MEASURES
URBANIZATION

Goal: To promote the orderly and efficient expansion of Newport's city limits.

Policy 1: The City of Newport will coordinate with Lincoln County in meeting the requirements of urban growth to 2031.

Implementation Measure 1: The adopted urban growth boundary for Newport establishes the limits of urban growth to the year 2031.

1.) City annexation shall occur only within the officially adopted urban growth boundary.

2.) The official policy shall govern specific annexation decisions. The city, in turn, will provide an opportunity for the county, concerned citizens, and other affected agencies and persons to respond to pending requests for annexation.

3.) Establishment of an urban growth boundary does not imply that all included land will be annexed to the City of Newport.

Policy 2: The city will recognize county zoning and control of lands within the unincorporated portions of the UGB.

Implementation Measure 2: A change in the land use plan designations of urbanizable land from those shown on the Lincoln County Comprehensive Plan Map to those designations shown on the City of Newport Comprehensive Plan Map shall only occur upon annexation to the city.

1.) Urban development of land will be encouraged within the existing city limits. Annexations shall address the need for the land to be in the city.

2.) Urban facilities and services must be adequate in condition and capacity to accommodate the additional level of growth allowed in the city's plans. Those facilities must be available or can be provided to a site before or concurrent with any annexations or plan changes.

Policy 3: The city recognizes Lincoln County as having jurisdiction over land use decisions within the unincorporated areas of the UGB.

Implementation Measure 3: All such decisions shall conform to both county and city policies.

1.) Unincorporated areas within the UGB will become part of Newport; therefore, development of those areas influences the future growth of the city. Hence, the city has an interest in the type and placement of that growth. Lincoln County shall notify the city of any land use decision in the UGB lying outside the city limits. The county shall consider recommendations and conditions suggested by the city and may make them conditions of approval.

2.) The city shall respond within 14 calendar days to notifications by the county of a land use decision inside the adopted UGB. The county may assume the city has comments only if they are received inside of that 14 days.

Policy 4: The development of land in the urban area shall conform to the plans, policies, and ordinances of the City of Newport.

Implementation Measure 4a: The City of Newport may provide water and wastewater services outside the city limits consistent with the policies for the provision of such services as identified in the applicable Goals and Policies of the Public Facilities Element of the Comprehensive Plan.

Implementation Measure 4b: Amendments to UGB Boundaries or Policies. This subsection delineates the procedure for joint city and county review of amendments to the urban growth boundary or urbanization policies as the need arises.

1.) Major Amendments:

a.) Any UGB change that has widespread and significant influence beyond the immediate area. Examples include:

(1) Quantitative changes that allow for substantial changes in the population or development density.

(2) Qualitative changes in the land use, such as residential to commercial or industrial.

(3) Changes that affect large areas or many different ownerships.

b.) A change in any urbanization policy.

2.) Minor Boundary Line Adjustments: The city and county may consider minor adjustments to the UGB using procedures similar to a zone change. Minor adjustments focus on specific, small properties not having significant impact beyond the immediate area.

3.) Determination of Major and Minor Amendments: The planning directors for the city and county shall determine whether or not a change is a minor or major amendment. If they cannot agree, the planning commissions for the city and county shall rule on the matter. The request shall be considered a major amendment if the planning commissions cannot agree.

4.) Initiation, Application, and Procedure: Individual or groups of property owners, agencies that are

affected, the planning commissions, or the city or county governing bodies may initiate amendments. Applicants for changes are responsible for completing the necessary application and preparing and

submitting the applicable findings with the application. The planning commissions for the city and county shall review the request and forward recommendations to the Newport City Council and the Lincoln County Board of Commissioners.

The city and county governing bodies shall hold public hearings on the request. Amendments become final only if both bodies approve the request.

5.) Findings shall address the following:

a.) Land Need: Establishment and change of urban growth boundaries shall be based on the following:

1.) Demonstrated need to accommodate long range urban population, consistent with a 20-year population forecast coordinated with affected local governments; and

2.) Demonstrated need for housing, employment opportunities, livability or uses such as public facilities, streets and roads, schools, parks and open space, or any combination of the need categories in this subsection;

b.) Boundary Location: The location of the urban growth boundary and changes to the boundary shall be determined by evaluating alternative boundary locations consistent with ORS 197.298 and with consideration of the following factors:

1.) Efficient accommodation of identified land needs;

2.) Orderly and economic provision of public facilities and services;

3.) Comparative environmental, energy, economic, and social consequences; and

4.) Compatibility of the proposed urban uses with nearby agricultural and forest activities occurring on farm and forest land outside the UGB.

c.) Compliance with applicable Statewide Planning Goals, unless an exception is taken to a particular goal requirement.

6.) Correction of Errors: Occasionally an error may occur. Errors such as cartographic mistakes, misprints, typographical errors, omissions, or duplications are technical in nature and not the result of new information or changing policies. If the Newport City Council and the

Lincoln County Board of Commissioners become aware of an error in the map or text of this adopted urbanization program, either body may cause an immediate amendment to correct the error. Both bodies must, however, agree that an error exists. Corrections shall be made by ordinance after a public hearing. The governing bodies may refer the matter to their respective planning commissions, but that is not required.

Policy 5: The city is responsible for public facilities planning within its urban growth boundary.

Exhibit C
Ordinance No. 2049
(File No. 3-CP-12)

FINAL

Big Creek Dam No. 1 and No. 2

Preliminary Geotechnical Investigation and Seismic Evaluation

City of Newport, Oregon

Prepared by:

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HDR Project No. 174693
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February 2013

the appropriate information needed for input to seismic response models of the CSZ events. The time histories developed and presented in Appendix D will be suitable for use during the next phase of evaluation.

Finalizing the CSZ ground motions early in the next phase of work will be an important step for the project as HDR's experience with the seismic response analyses recently completed at Reclamations nearby Scoggins Dam has shown that the CSZ hazard will control the site response and safety of the dam. Currently, available information suggests that the CSZ earthquake events can have very large durations (100 to 400 seconds) and there can be significantly different remediation concepts and costs associated with this range of ground shaking durations. It is anticipated that the new information from PEER will increase the confidence in the ground motions used for evaluation and design and to help justify the shortest ground motion duration that is reasonable for the site.

6.3 Other Geologic Hazards

Given the location of BC No.1 and BC No. 2 near the Oregon coast and within the Oregon Coast Range, the geologic hazards of Tsunami inundation and landslides are possible. However, the Tsunami inundation hazard map (Figure 1) shows the downstream toe of the lower dam east and outside of the inundation line indicating that inundation during a tsunami is not likely to occur. A review of the State Wide Landslide Information Map produced by Oregon Department of Geology and Mineral Industry (DOGAMI, 2012) (http://www.oregongeology.org/slido/index.html) shows two landslides within the last 16 years within 1 kilometer of the dam sites. In addition, a large area of highly erodible Quaternary material is mapped adjacent to and north of the dam sites. This area has the potential for producing large volumes of sediment during periods of heavy rainfall. An existing or nascent landslide has the greatest potential to affect the stability of the dams if it occurs within any of the abutments. Another geologic hazard is the presence of liquefiable soils. Non-cohesive silts and silty sands are known to exist in the foundation at both sites. These materials, where they exist, are subject to liquefaction under seismic loading as discussed in Section 6.2.

7.0 SUBSURFACE CONDITIONS

7.1 Subsurface Stratigraphy

BC No. 1

As discussed in Section 4.0, a series of explorations were performed at the BC No. 1 dam site including: one boring and two CPTu soundings from the embankment crest, two CPTu soundings near the downstream toe, and a seismic refraction geophysical survey line across the crest of the dam from abutment to abutment. As previously noted, the seismic refraction surveys were of limited value and not included in development of the subsurface model at the BC No. 1 dam site. As shown on boring log BC1-B-1 in Appendix B.1, clayey silt (MH, defined as elastic silt with high plasticity) embankment fill was encountered from just beneath the dam crest (EL 45.4) to EL 23.5 feet. The embankment fill is underlain by sandy silt and clayey silt (EL 23.5 to -4.6 feet), and silty sand alluvium (EL -4.6 to about EL -37.6 feet) where weathered bedrock consisting of decomposed siltstone was encountered. Unless otherwise indicated, all elevations noted in this report are NAVD88.

Siltstone bedrock outcrops north and south of the embankment dam abutments. Based on the results of the boring, and CPTs (summarized further below), a general concept for a geologic model of the BC No. 1 site was developed. Using this concept, a typical cross-section through the maximum section of the dam was developed and is shown on Figure 3. A subsurface profile along the alignment of the crest of the dam is shown on Figure 4.

Following is a description of the materials (in accordance with the USCS ASTM D2487) encountered in boring BC1-B-1 and drilled from the crest of the dam. It should be noted that the embankment and foundation soils found at the site appear to be similar to materials of volcanic origin and hence display some unusual characteristics (i.e., high void ratio and water contents, moisture contents in excess of the liquid limit) These characteristics are not necessarily indicative of problematic soils but of the need for proper handling, testing, and evaluation procedures as the project progresses through future evaluation and construction phases.

Clayey SILT with some Sand (Dam Fill): The dam fill material generally consists of low to medium plasticity clayey silt with some fine sand. As discussed in Section 3.0, the plans for the original dam construction in 1951 indicate up to 21 feet of clayey silt fill was placed to construct the embankment. This is consistent with the conditions found in boring BC1-B-1 where fill appeared to extend from EL 47.4 to EL 23.9 feet (23.5 feet below the crest of the dam). SPT N-values ranging from 0 to 4 indicate the relative consistency of the fill is very soft to soft. Results of laboratory index testing on selected samples showed a plasticity index (PI) ranging from 20 to 28 (MH), water contents near the liquid limit (LL), and a fines (silt and clay) percentage near 50 percent.

Sandy SILT with some Clay (Alluvium): Alluvial material consisting of low to medium plasticity sandy silt with fine sand was encountered in BC1-B-1 below the dam fill, extending to EL 17.4 feet (depth of 30 feet). SPT N-values ranged from 0 to 5, indicating the relative consistency of the alluvium is very soft to medium stiff. Results of laboratory index testing on selected samples showed a PI of 14, LL of 49 which is a borderline low to high plasticity silt (ML-MH), water content above the LL, and fines percentage of 62 percent.

Clayey SILT with some Sand (Alluvium): This material was encountered from EL 17.4 to -4.6 feet (depth of 30 to 52 feet). Atterberg limit testing results showed this silt has a PI ranging from 14 to 41 (MH), LL

ranging from 54 to 87, and water contents at or slightly below the liquid limit. The SPT N-values recorded in this layer ranged from 0 to 2, indicating the soil is very soft to soft.

Silty SAND (Alluvium): Alluvial material consisting of low plasticity silty sand with isolated lenses of medium plasticity sandy silt and organic silt was encountered beneath the clayey silt from EL -4.6 to EL -37.6 feet (depth of 52 to 85 feet). SPT N-values ranged from 0 to 3, indicating the relative density is very loose. Laboratory testing indicates these soils generally have low plasticity with PI ranging from 0 (non-plastic) to 8 (ML) with few layers ranging from 22 to 28, LL ranging from 42 to 57, and fines percentage ranging from 22 to 53 percent. Scattered organics and wood chips/debris were encountered throughout this layer.

Siltstone (Marine Sedimentary Rock): The boring terminated in decomposed to weathered siltstone. In the decomposed condition, the siltstone consisted of stiff to hard, clayey silt. Results from the CPT penetrations also suggested that decomposed to weathered siltstone was encountered providing a basis to estimate the bedrock surface profile at the BC No. 1 site. The elevation of the siltstone layer that was found in each of the exploration borings or CPT soundings is summarized in Table 4 and shown on Figure 3 and Figure 4. The elevation of the siltstone layer varies from -16 to -38 feet with the lowest elevation near the original creek channel and highest siltstone elevation (i.e., shallowest) occurring beneath the northern and southern ends of the dam. Siltstone bedrock outcrops north and south of the embankment dam were identified in the field and surveyed with a handheld Global Positioning System (GPS) unit.

Soil samples are not obtained with a CPTu sounding; therefore it is generally accepted practice to establish a correlation between at least one soil boring and CPTu soundings during site characterization investigations. BC1-CPT-3 was performed adjacent to boring BC1-B-1 (see Figure 9) to allow a correlation of the CPTu data with the soil boring data, and to use this correlation to interpret the results from the other three CPTu soundings at the BC No. 1 dam site. The correlation with the soil boring is required primarily to determine if the CPTu derived soil classifications (i.e., sandy or clayey soils) match the soil classifications determined from visual classification and laboratory soil sample index testing. SPT N-values measured in the boring also can be compared to the CPTu data as well as laboratory measured undrained shear strength (S_u) values to develop a site specific correlation between both SPT and CPT measurements, and the shear strength of the embankment and foundation soils.

For seismic response evaluations, it is important to delineate materials that may be subject to liquefaction verses those that may soften due to cyclic loading. This is typically done by identifying materials that will behave as "sand-like" (potentially liquefiable) from those that will behave as "clay-like" (potentially susceptible to cyclic softening). For purposes of this study, the recommendations of Boulanger and Idriss (2004), and Bray and Sancio (2006) were used to identify these behavior characteristics. The primary soil property used for this characterization is the soil PI. The percentage of silt/clay in the soil matrix is also a consideration in this designation. As discussed in Section 7.0, "sand-like" soils generally have a PI less than 7 and may be potentially liquefiable. "Clay-like" soils generally have a PI equal to or greater than 7 and may be potentially susceptible to cyclic softening. A minimum fines content of between 35 and 50 is also considered for the "clay-like" designation.

Soil categorization based on a specific PI value (i.e., 12) and consideration of fines content is not possible without laboratory soil sample testing. For the purpose of the preliminary seismic evaluation, an attempt was made to use the CPTu soundings to classify soils as "clay-like" and "sand-like". Additional soil borings and laboratory testing will be required during future study phases and design to determine the PI of the soils and the appropriate soil behavior characteristics during and immediately following an earthquake.

Identification of potentially liquefiable soils that are non-plastic or have low plasticity from more plastic soils using cone penetrometer test data generally can be established using the soil behavior type index. Robertson and Wride (1998) developed this method specifically to evaluate the liquefaction potential of soils based on CPT data. Based on their method, soils are considered to have liquefaction potential if the soil behavior type index (I_c) is less than 2.6. With this method, specific PI values for the soil are not addressed.

The I_c profile for BC1-CPT-3 is plotted on Figure 11. The I_c values are generally greater than 2.6 below EL 39 feet (depth of 8 ft); therefore, based on this method, the soils should not be potentially liquefiable. However, based on the laboratory index testing results and evaluation of the boring BC1-B-1 drilling log, the silty-sand soils from EL -5 to -37 feet are primarily non-plastic or have a low PI (<= 7), have less than 35 to 50 percent fines, and should be considered potentially liquefiable. As shown on Figure 11, the I_c values from the CPTu are about 3 to 3.2 for the silty sand layer. In fact, the I_c values in the silty sand layer are not appreciably different from the I_c values for the medium plasticity clayey silt soils in BC1-B-1 between EL 20 and -4.6 feet. Based on this comparison, I_c does not appear to be a good indicator of liquefiable sand-like soil versus non-liquefiable clay-like soil for the soils encountered at the BC No. 1 dam site. Therefore, I_c was not used as a means to identify soils that are potentially liquefiable (PI(<= 7) at this time. As previously noted, the foundation alluvial soils have some unusual characteristics that are similar to materials associated with materials that originate from volcanic ash. I_c will continue to be considered during future investigation to identify any adjustments that are appropriate for a potential liquefaction designation in the seismic response evaluations

For this project, a simple methodology was established to delineate sand-like soils from clay-like soils by comparing the CPTu cone resistance (q_t) to the normalized pore pressure ratio (B_q). This method only provides an estimate for this preliminary seismic evaluation and additional borings and laboratory testing will be required to accurately delineate soils with a PI less than or greater than 7. As shown on Figure 12, generally when the q_t values were relatively low and uniform during penetration through the very soft to soft MH soils and the B_q was positive, the soils had a higher plasticity as confirmed by Atterberg limit testing of the samples from boring BC1-B-1 (Appendix C.1). There was a discrepancy between the interpretation using this method and boring BC-B-1 between EL +5 and -5 feet. In this interval, the CPTu interpretation would indicate the soils are sand-like, but the laboratory testing indicated the soils were an MH with a PI greater than 7. To be conservative, soils below an elevation of 0 feet were considered as potentially liquefiable in our post-earthquake stability analyses.

This technique was applied to each CPTu profile and the results are shown on Figure 13 and Figure 14 for BC1-CPT-1 and BC1-CPT-4, respectively. Thin apparently sand-like soil layers that occurred within the clayey layers were not differentiated if the sand-like layers were thinner than about 5 feet. The same criterion was applied for thin clayey layers that occurred within a sandy layer.

The q_t and B_q values for BC1-CPT-4 are considerably different from the BC1-CPT-3 profile; the CPTu soundings are approximately 100 feet apart along the crest of the dam. The q_t for BC1-CPT-4 below about EL 0 feet is much less than encountered in BC1-CPT-3. Also, the B_q values are relatively high for BC1-CPT-4 compared to negative values for BC1-CPT-3. The proximity of BC1-CPT-3 and BC1-B-1 to the original creek channel may explain why these materials appear to be sand-like as compared to BC1-CPT-4.

The results of this evaluation and the stratigraphy interpreted from the explorations are summarized in Table 5. The CPTu soundings indicate the delineation of sand-like and clay-like soils vary across the dam site. For this preliminary seismic evaluation, the soil profile for BC1-B-1/BC1-CPT-3 and the interpreted soil profile for BC1-CPT-4 were used for the seismic evaluation and geotechnical analyses. For the BC1-

B-1/BC1-CPT-3 profile, an elevation of 0 feet was selected for the top of the potentially liquefiable silty sand layer.

BC No. 2

A series of explorations were also performed at the BC No. 2 dam site; three borings, three CPTu soundings, and two seismic refraction survey lines. Two of the borings and the three CPTu soundings were performed on the embankment crest. Boring BC2-B-3 was performed near the downstream toe of the embankment. As previously noted, the seismic refraction survey results were of limited value and not used in the development of the subsurface model at the BC No. 2 dam site.

About 67 feet of MH embankment fill was encountered to EL 24.6 feet in boring BC2-B-1. About 42 feet of silty sand (SM) and clayey high plasticity silt (MH) embankment fill was encountered to EL 49.2 feet in boring BC2-B-2. These two borings confirmed information presented on the 1968 construction drawings and preliminary design report for the dam modifications (CH2MHill, 1974), indicating that the alluvium was removed to the top of weathered siltstone bedrock for the construction of the cutoff trench as shown on Figure 8.

A typical cross-section through the dam and foundation compiled from the available design and exploration information obtained during this study is shown on Figure 7. The location of this cross-section is shown on Figure 10. Upstream and downstream of the cutoff trench, the embankment fill is probably underlain by alluvium as represented by the foundation soils encountered in boring BC2-B-3. In general, HDR believes that the embankment fill and alluvial sediment are underlain by decomposed to weathered siltstone bedrock encountered in the borings, CPT soundings, and outcrops north and south of the embankment dam.

The following is a description of the materials encountered in boring BC2-B-1. These descriptions, excluding the reference elevation information, are similar to the materials found in boring BC2-B-2:

Clayey SILT with some Sand (Dam Fill): The dam fill material generally consisted of high plasticity clayey silt with some fine sand that extends to EL 26.6 feet, 65.0 feet below the crest of the dam. The fill is generally stiff to very stiff with typical SPT N-values of 10 to 13; however, lower N-values were obtained to a depth of about 15 feet below the crest of the dam and in the bottom 10 feet of the fill. Laboratory testing on two samples indicates a PI of 10 to 18 (MH), with a water content below the liquid limit.

Silty Sand (Fill): A 2-foot-thick layer/lense of nonplastic silty fine sand was found in the BC2-B-1 embankment fill between EL 26.6 and 24.6 feet. An N-value of 2 indicates the relative consistency of this fill material is very loose.

Siltstone (Marine Sedimentary Rock): Decomposed Siltstone (Clayey silt) was encountered from EL 24.6 feet to the boring termination at EL 11.6 feet. From EL 24.6 to 19.6 feet, the decompressed siltstone is hard with N-values of 30 and 32. The siltstone could be sampled with rock coring methods from EL 19.6 to 11.6 feet. The bedrock in the core samples was generally highly weathered and for the two core runs completed were 100 and 93 percent, respectively.

In boring BC2-B-3 drilled near the downstream toe of the embankment, the following foundation alluvium materials were encountered:

Silty SAND to sandy silt with some clay (Fill): The fill extended to a depth of 10 feet (EL 40 feet). It was unclear whether this fill was placed as part of the original construction or as part of a later dam

modification in 1976. The SPT tests in this layer showed the fill is loose to medium dense with SPT N-values ranging from 4 to 14. Laboratory testing of two samples indicated a USCS designation of ML/SM with a PI ranging from 12 to 14. The fines percentage ranged from 48 to 52 percent. Since the PI is greater than or equal to 7, the material was classified as clay-like for the seismic analyses.

Sandy SILT and Silty SAND (Alluvium): The sandy silt (MH) and silty sand (SM) extended 20 feet below the base of the fill to the surface of decomposed siltstone at EL 20 feet and is generally loose with SPT N-values ranging from 2 to 9. The soil generally has 35 to 64 percent fines content and a PI ranging from non-plastic (i.e., sand-like) to 19.

Siltstone (Marine Sedimentary Rock): Decomposed siltstone extended from EL 20 feet to the termination of the boring at EL 8.6 feet. The siltstone had a stiff consistency and gradationally classified as a borderline ML/MH to SM material. There were some scattered gravel and wood fragments in the siltstone.

7.2 Engineering Property Characterization

The following sections summarize the engineering properties of the embankment and foundation soils/bedrock that are required to assess seepage conditions and associated water pressures and gradient in the dam and foundation, along with the potential for liquefaction or cyclic strength degradation and the corresponding shear strength values to be used in slope stability analyses.

Basic Soil Parameters

The basic soil parameters summarized in Table 6 were developed for input to the geotechnical analyses including the total unit weight and volumetric water content.

Permeability (K)

An estimate of the steady-state seepage phreatic water surface through the dam and foundation is required for stability and seismic response evaluations. To estimate the location of the phreatic surface, the vertical permeability (K_v), horizontal permeability (K_h), and the ratio of vertical to horizontal permeability of the embankment and foundation soils at the site are required. Laboratory permeability tests were not performed for this preliminary seismic response evaluation of the Big Creek Dams. Instead, permeability values were selected for the different soil types included in the models based on a variety of published sources of information including values developed through extensive testing for major levee improvements in the Sacramento River basin near Sacramento, California (Board of Senior Consultants [BOSC], 2010). A summary of estimated permeability values for a wide range of soil types adopted for these evaluations are shown in Table 7. The suggested model layer colors also shown in this table were established to provide for consistency in presentation of model layer characteristics as the project progresses.

The soil classifications and fines content determined from laboratory testing of samples obtained from the borings completed at BC No. 1 and BC No. 2 are summarized in Tables C.1-1 and C.1-2, respectively (Appendix C). As noted above, the foundation soils at both sites are predominantly high plasticity silt (MH) and silty fine sand (SM). Embankment materials are predominantly MH materials. In addition to the soil materials in the embankment and foundation, there is a blanket drain in both dams. A review of the available construction documents found that there were no specifications for this material. Further, blanket drain materials were not sampled during the recent site exploration program. For the analyses, HDR has assumed that the blanket drains were constructed from slightly silty fine sand (3 to 7 percent fines).

A summary of the selected permeability values and K_v/K_h ratios are presented in Table 8. In addition to these presumptive values, permeability values were also estimated based on CPTu pore pressure dissipation tests. One dissipation test performed in BC1-CPT-3 at a depth of 39.7 feet indicated a K of $5x10^{-8}$ centimeters per second (cm/sec) in the clayey silt material and a test in the silty sand material at a depth of 59.7 feet yielded a value of $3x10^{-7}$ cm/sec. These values are lower than the typical values summarized in Table 7 and Table 8, and hence were selected as the lower bound values used in the analyses.

Soil Strength Parameters

Shear strength parameters for the existing static (pre-earthquake) and post-earthquake loading conditions were selected for each soil type in the typical BC No. 1 and BC No. 2 cross-sections shown on Figure 3 and Figure 7, respectively. For BC No. 1, the static and post-earthquake strength parameters were developed from interpretation of the CPTu data, laboratory testing, and correlations with soil index properties. For BC No. 2, the strength parameters were based on the interpretation of CPTu data, SPT N-values, and strength data included in the CH2MHill preliminary design report (1974).

As discussed further in Section 8.0, below, an evaluation of the SPT $N_{1,60}$ values and liquefaction potential of the sand-like soils at both dam sites indicates that SM and ML materials will liquefy due to an earthquake on either the Yaquina faults (M6.1) or CSZ (M9.0). These materials have reasonably good strength under static loading conditions, however, they will lose significant strength during an earthquake event. Similarly, there will be cyclic softening and loss of strength of some of the "clay-like" MH embankment and foundation soils during and immediately following either earthquake loading condition.

BC No. 1 Dam

For BC No. 1 dam, information from boring BC1-B-1 and the four CPTu soundings were used to assess the static and post-earthquake shear strength of the soils used in stability evaluations as summarized below.

Static Shear Strength. Estimated minimum factors of safety (FOS) for the static loading condition (long-term steady state seepage conditions), were performed using estimates of drained (effective stress) strength parameters (e.g., USACE, 2003). The effective stress friction angle for the clayey-silt soils were estimated based on laboratory PI determinations (Mitchell, 1976). For an average PI of 30 for the clayey silt embankment soils in BC1-B-1, a drained friction angle of 28 degrees was selected. For the silty sand foundation soils in boring BC1-B-1, the drained friction angle was estimated using equivalent $N_{1,60}$ values estimated from the CPTu profiles. For an average $N_{1,60}$ of 4 blows per foot (bpf), a drained friction angle of 28 degrees was also estimated (Mayne et al, 2001). A cohesion of 0.1 kips per square foot (ksf) was included for both the embankment and foundation soils to reflect the expected curvature of the failure envelope in the low effective stress range and minimize the influence of shallow (infinite slope) failure surfaces on the estimates of the location and minimum FOS during stability analyses. A summary of the drained shear strength parameters used for BC No. 1 static stability evaluations is presented in Table 9.

Post-Earthquake Strength. Post-earthquake strengths were developed in a two-step process. First, a general determination was made on an expected "sand-like" or "clay-like" behavior as previously discussed. For those embankment and foundation materials that are expected to have a "clay-like" behavior, estimates of the peak undrained shear strength (S_u) of the embankment and foundations soils were made based on the results from the CPTu tests (see Figure 15 and Figure 16). Using the estimates of peak strength and results of laboratory cyclic simple shear tests, an estimate of the amount of strength degradation was made to establish the "post-earthquake" shear strength input to the stability analysis models. For the foundation materials that are estimated to have a more "sand-like" response to

earthquake loads, the post-earthquake residual strength (also referred to as post-earthquake steady state strength) for the potentially liquefiable sand-like soils was estimated using the relationship proposed by Seed and Harder (1990) as shown on Figure 17. Seed (2010) calculated a least squares fit through the Seed and Harder (1990) data, and this relationship (red dashed curve) was used to estimate the post-earthquake strength of the sand-like soils (PI<7). The CPTu derived $N_{1,60}$ values adjusted for fines content were used with the Seed and Harder (1990) relationship to estimate the post-earthquake undrained strength as shown on Figure 18 for BC1-CPT-3. A value of 0.2 ksf (200 pounds per square foot) was selected for the post-earthquake stability analyses of BC No. 1.

As shown on Figure 15 and Figure 16, shear strength values four MH embankment and foundation materials encountered in the BC1-CPT-3 and BC1-CPT-4 soundings were estimated using the CPTu q_t values and a cone factor (N_k) of 15. N_k can vary from about 10 to 20; however, a value of 15 is typically used for estimating the shear strength for these soil types (Robertson, 2009). The interpreted S_u values for BC1-B-1/BC1-CPT-3 and BC1-CPT-4 are summarized in Table 10 and Table 11, respectively.

The interpreted undrained shear strength for both the BC-1 soundings generally decreased with depth. The S_u value for the embankment fill is about 1 ksf. For BC1-CPT-3, the S_u value decreases to about 0.75 ksf and for BC-CPT-4 it decreased to about 0.5 ksf. The S_u values below EL -25 feet for BC1-CPT-4 were considerably less than what would be expected for a normally consolidated soil with an S_u/S'_v ratio of 0.22 (S'_v is the vertical effective stress) and a normal range of void ratio and corresponding effective stress. The S_u/S'_v ratio is based on an average PI of 30 for the MH soils in BC1-B-1. This relatively low strength however, is reasonably consistent with the high void ratios (low unit weights) encountered, particularly in the foundation soils at the site. The relatively high normalized pore pressure ratios and low q_t values for BC1-CPT-4 (Figure 14) may indicate some influence of an artesian groundwater pressure near the top of the siltstone layer.

For the clayey silt soil, results from the laboratory static and cyclic simple shear tests were used to develop strength reduction factors to apply to the insitu CPTu strengths to account for the loss in strength due to cyclic loading. The result for the cyclic simple shear test for the undisturbed clayey silt soil sample from BC1-B-1 is shown on Figure 19. The test was performed at a cyclic strength ratio of 0.8 and the sample failed after 27 cycles of loading. As shown, the test result agrees with the published data presented by Boulanger and Idriss (2007).

Immediately after completion of the cyclic test, a monotonic simple shear test was performed to determine the post-cyclic undrained shear strength. This test showed that the undrained shear strength of the clayey soil was reduced by 33 percent due to the effects of cyclic loading. Therefore, the S_u profiles shown on Figure 15 and Figure 16 were reduced by 33 percent to account for the effect of cyclic loading; these values are included in Table 10 and Table 11 for profiles from BC1-CPT-3 and BC1-CPT-4, respectively.

BC No. 2

Static Shear Strength. As discussed in Section 7.1, the soils for BC No. 2 consisted of the clayey-silt fill soil within the embankment and cut-off trench and the alluvial soils outside the cut-off trench as represented by boring BC2-B-3. Estimated minimum FOS for the static loading condition (long-term steady state seepage conditions), were also performed using estimates of drained (effective stress) strength parameters (e.g. USACE, 2003). Estimates of the drained shear strength properties for the various embankment and foundation soils were obtained from the CH2MHill 1974 preliminary design report and are summarized in Table 12. A conservative value of 35 degrees was assumed for the gravel filters and a relatively low total unit weight of 82.4 pounds per cubic foot (pcf) with zero strength was assumed for the approximate 15 foot thickness of reservoir sediment.

Post-Earthquake Strength. The undrained shear strength parameters used as part of the CH2MHill 1974 preliminary design are shown in Table 13.

The estimated peak undrained shear strength based on three CPTu sounding results are shown for "clay-like" soils on Figure 20 through Figure 22. The interpreted values are somewhat erratic; however, the undrained shear strength values are generally between 1 and 3 ksf.

The post-earthquake strength values used for BC No. 2 were selected based on the results of the liquefaction and cyclic softening analyses discussed in Section 8.0, below. As shown in Table 13, the post-earthquake undrained shear strength for the clay-like embankment dam soils soundings was reduced to 66 percent of the pre-earthquake strength if the factor of safety against cyclic softening was less than 1.2. For boring BC2-B-3, post-earthquake residual undrained (steady state) shear strength was calculated for the liquefiable sand-like soils based on SPT blowcounts as described for BC No. 1.

8.0 SEISMIC HAZARDS AND GROUND MOTIONS

As previously noted in Section 6.2, above, a seismic hazard evaluation including the identification of representative of ground motions for the dam sites was performed as part of these studies (Cornforth, 2012) ~~and is~~ included in Appendix D. Specifically, this portion of the current study included the following:

- Identification of the principal seismic sources that contribute to the seismic hazard,
- Development of site specific response spectra,
- PSHA to identify peak ground accelerations as a function of recurrence interval for the identified seismic sources, and
- Identification of representative time histories for the identified seismic sources to use in seismic response evaluations.

8.1 Seismic Sources

The primary seismic sources identified that could impact the dam sites are the shallow crustal earthquakes within the North American tectonic plate and the CSZ. As shown in Table 1 of the Cornforth (2012) report, the Yaquina fault located 2.4 km (1.5 miles) from the site can generate a M 6.1 earthquake and the CSZ located about 24 km (15 miles) can generate a megathrust M 9.0 earthquake. These hazard sources are applicable to both dams since the distance of the sources to the dams is similar.

Several earthquakes about M 4.9 or smaller have occurred in the vicinity of the Big Creek dams in the last 170 years. In addition, recent research has strongly suggested a notable estimated M 9.0 megathrust (interface) earthquake event that occurred around January of 1700 on the CSZ.

8.2 Probabilistic Seismic Hazard Analysis (PSHA)

A PSHA was performed to develop estimates of peak ground motions at the dam sites that correspond to return periods of 475 to 2,475 years utilizing the USGS 2008 Interactive Deaggregation's web site. As shown in Table 2A of the Cornforth report, the CSZ would contribute 67 percent and the Yaquina fault 33 percent to the PGA hazard (0.0 second) for an earthquake with a return period of 2,475 years. Based on the USGS deaggregation, the magnitude and distance for the principal seismic sources are provided in Table 1 (all tables are provided at the end of this report):

8.3 Ground Motions

A number of factors need to be considered in the selection of the ground motion return period for safety evaluations and design including: regulatory requirements, potential loss of life, economic damage, and the need to maintain water supply after the seismic event. For purposes of these evaluations, ground motions for a 2,475-year return period were selected for the initial seismic evaluation of the BC No. 1 and BC No. 2 dams; this corresponds to a 2 percent probability of exceedance for a 50-year time interval.

The deaggregated earthquake ground motion hazards determined from the analysis for a 2,475-year return period and the corresponding PGAs are shown in Table 2.

The PGA values were determined using attenuation relationships applicable to each seismic source. The 84th percentile ground motion corresponds to the value that is one standard deviation above the mean value. For the Yaquina fault source earthquake, this resulted in estimated PGA values of 0.52g to 1.10g

for the different attenuation relationships with an average of 0.83g for a M 6.1 reverse fault rupture event. For the CSZ interface-megathrust source, four attenuation relationships were used and a weighted average was applied to estimate the 0.56 PGA value that would occur in the 0.4- to 2-second period range. The average response spectra for the 2,475-year return period are shown in Figure 23 and Figure 24 for the Yaquina and CSZ seismic sources, respectively.

8.4 Ground Motion Time Histories

Available records were searched to select appropriate ground motion time histories that can be used in explicit seismic response evaluations. The selection of an appropriate time history is typically based on similar geologic conditions, earthquake magnitudes, fault mechanism, and distance to fault rupture. The selected time histories were provided in Excel format and accompanied the Cornforth (2012) report. For the CSZ earthquakes, a limited database of ground motions are available; however, as previously noted, numerous seismic records from the recent Tohoku, Japan, and Chili subduction zone earthquake are being evaluated by the PEER. This is important because the duration of intense ground shaking during a CSZ event is uncertain and evaluation of time histories from a similar subduction type earthquake will improve this understanding and the basis for updated safety evaluations and design.

9.0 SEISMIC RESPONSE

9.1 Evaluation Procedure

Evaluating the potential response of embankment dams to significant ground shaking events is a complex process and requires an understanding of the seismic hazard, site characteristics, and the corresponding material properties of the embankment and foundation relative to static and seismic loading conditions as discussed in the preceding sections of this report. Experience has shown that the most difficult aspect of predicting the response of structures to seismic loading is characterizing the shear strength of foundation and embankment materials, particularly if they are of low density (contractive) and subject to the loss of strength under rapid loading conditions that are typical during large earthquake events.

The standard of care for completion of seismic response evaluations generally consists of a series of increasingly complex site investigations, laboratory testing, and seismic response evaluations. Initial evaluations tend to be more conservative. If these initial evaluations determine that the structures will respond favorably to seismic loads, safety evaluations can be terminated with relatively simple and inexpensive evaluations. However, if the initial (and simplified) evaluations identify potential safety concerns or deficiencies, supplemental site characterization and seismic response evaluations are typically undertaken to reduce the conservatism of the simplified evaluation procedures. Supplemental investigations and evaluations typically result in either; 1) elimination of safety concerns, or 2) minimization of the safety modification requirements and costs should a deficiency be confirmed.

The simplified evaluation completed for this initial evaluation of the Big Creek Dams consisted of the following:

1. Development of simplified geologic model of the sites including representative dam axis profiles and cross-sections for engineering evaluation (Sections 2 through 7).
2. Identification of the seismic hazards at the site (Section 6.2 and 8.0)
3. Estimation of engineering properties including permeability and shear strength of the various embankments and foundation materials in the cross-section models (Section 7.2).
4. Estimation of any shear strength reduction that may occur during and/or immediately following and earthquake due to liquefaction (typical of loose, contractive "sand-like" material behavior), or cyclic softening (typical of low density, and saturated "clay-like" material; Section 0).
5. Perform steady state seepage and stability analyses using estimated water pressures and drain shear strength properties to estimate minimum static FOS for each dam (Section 10.1).
6. Perform "post-earthquake" stability analyses using any appropriate strength reduction to estimate minimum "post-earthquake" stability FOS (Section 10.2).

Results of the initial site characterization including *insitu* testing, laboratory testing, evaluation of the material characteristics including seepage and shear strength properties of the embankment and foundation materials at each site along with the potential for shear strength reduction have been discussed in previous sections of this report. In the sections that follow, results of additional evaluations of strength reduction potential, particularly of the high plasticity clayey silts found in the dams and dam foundations are presented. The initial site characterization included limited direct sampling and testing for correlation to CPTu results. The one set of cyclic direct simple shear laboratory test results showed cyclic softening and strength reduction. Further evaluation of the CPTu tests discussed below support estimates of strength reduction that may occur in the "clay-like" embankment and foundation soils at the site.

Finally, the results of the steady state (static) stability, and post-earthquake stability analyses (using the estimates of shear strength reduction due to liquefaction or softening) are presented in Section 10.

In a simplified evaluation procedure, the overall safety of the dams is assessed based on the estimated minimum stability FOS under both static and "post-earthquake" conditions. The minimum required FOS under static loading conditions are well established and documented under state and federal dam safety guidelines. In general, a minimum static factor of safety of 1.5 is required for significant and high hazard dams. Guidelines for "post-earthquake" FOS are more variable under state and federal safety guidelines. However, minimum "post-earthquake" FOS values are generally interpreted as follows:

1. Values that are less than 1.0 are indicative of a significant potential for a flow failure of the structure.
2. Values between 1.0 and 1.2 are generally indicative of a potential for large structure deformations. For this condition, additional seismic response evaluations using empirical to advanced numerical modeling methods will likely be required to assess potential deformations, available freeboard following an earthquake, and the potential for either an overtopping or seepage (through cracks) potential failure mode development.
3. Values greater than 1.2 are generally acceptable except for special conditions which may require further evaluation. Such conditions may include dams with limited available freeboard, long duration earthquakes (such as the CSZ events) that may produce abnormally large deformations, or unusual site or design conditions (steep abutments) where cracking could result in development of a failure mode even for relatively small deformations.

9.2 Cyclic Softening Evaluation Methodology

Boulanger and Idriss (2006) state that soils with a PI less than 7 may be susceptible to liquefaction while Bray and Sancio (2006) state that soils with a PI less than 12 is susceptible to liquefaction. Bray and Sancio include an additional condition that the ratio of water content to liquid limit should be greater than 0.85 for the soils to be susceptible to liquefaction. For purposes of this study, materials with a PI less than 7 were considered as sand-like and potentially liquefiable. All other soils with a PI greater than 7 to 12 were considered as subject to cyclic softening.

A discussion of the materials in the dams and foundations that are "sand-like" and may be subject to liquefaction have been presented in Section 7.0. The associated drained and undrained "post-earthquake" residual (steady state) shear strength for these materials have been estimated based on direct insitu SPT testing or indirect correlations between CPT and SPT blowcounts normalized to an overburden pressure of 1 ton per square foot, a hammer efficiency of 60 percent, and corrected for fines content ($N_{1,60}$). A comparison of the SPT $N_{1,60}$ values from the soil boring BC1-B-1 or $N_{1,60}$ values based on the CPTu q_t profile in BC1-CPT-3 is shown on Figure 25, Results for BC No. 2 including boring BC2-B-3 are presented in Appendix E. No further evaluations of the sand-like materials were performed to support the estimates of post-earthquake strength reduction that may occur.

For clay-like soils, the potential loss in strength was evaluated using the methodology proposed by Boulanger and Idriss (2007). Their method is based on the original simplified procedure by Seed and Idriss (1982) for estimating cyclic stress ratio (CSR) and comparing this value to the cyclic resistance ratio (CRR) to estimate a factor of safety (FOS) against cyclic softening (also liquefaction) where:

$$FOS = CRR/CSR$$

An FOS less than one indicates softening could occur.

The CSR is used to quantify the stresses that may develop insitu during cyclic earthquake loading based on the following equation:

$$CSR = 0.6*(a_{max}/g)*(S_{vo}/S'_{vo})*r_d*K_0*K_a$$

a_{max} = peak ground acceleration

g = acceleration of gravity

S_{vo} = Total overburden stress

S'_{vo} = Effective overburden stress

r_d = stress reduction coefficient

K_0 = Overburden stress correction factor

K_a = Ground slope correction

The CSR values were calculated using the PGA values determined for the Yaquina M 6.1 and CSZ M 9.0 deaggregated earthquake motions.

The procedure also requires an estimate of the CRR of the soils. To estimate CRR, first an estimate of the $CRR_{(M=7.5)}$ for clay-like soil is made from the following equation:

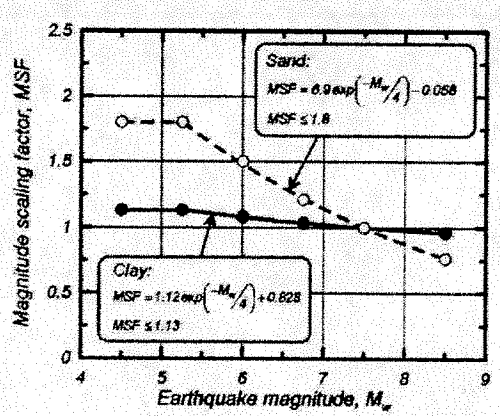
$$CRR_{(M=7.5)} = 0.8 * S_u/S'_{vo}$$

The CRR value is then adjusted for the earthquake magnitude as follows:

$$CRR = CRR_{(M=7.5)} * MSF$$

MSF = Magnitude scaling factor

The MSF is estimated based on the graph provided below. As can be seen, the MSF values for clay-like soils are less dependent on earthquake magnitude than sand-like soils.



9.3 BC No. 1

The FOS against cyclic softening for BC1-CPT-4 is shown on Figure 28. The FOS is acceptable to a depth of about 15 feet within the embankment, but decreases significantly in the relatively soft clay-like alluvial soils. This was expected based on the relatively low undrained shear strength values derived from the CPTu profile.

9.4 BC No. 2

The FOS against cyclic softening for BC2-CPT-1 is shown on Figure 29. The upper part of the embankment appears to be acceptable, but the lower portion above the siltstone has a relatively low factor of safety.

10.0 EMBANKMENT SEEPAGE AND SLOPE STABILITY ANALYSIS RESULTS

10.1 Embankment Seepage Analysis Results

The seepage analyses of BC No. 1 and BC No. 2 were performed using the finite element GeoStudio 2007 version 7.17 computer program. The purpose of these analyses was to estimate the location of the phreatic surface in the steady-state condition for use in slope stability and for yield acceleration analyses. To obtain the sensitivity of the phreatic line to the hydraulic conductivities, the seepage analyses were performed for the combination of the lower bound and upper bound permeabilities (referred to as hydraulic conductivity in Appendix F) of the foundation and embankment materials.

Analysis Cases

The seepage analyses were performed for the idealized cross-sections based on the results of CPTu borings BC1-CPT-3 and BC1-CPT-4, and geotechnical boring BC2-B-1, as previously discussed. The long-term or steady state seepage study cases are presented in Table 15. Due to the uncertainties in the functionality of the buried toe drain at BC No. 1, the seepage analysis was conducted for two cases of with and without toe drain. The toe drain for the BC No. 2 was assumed to be functional. A more detailed presentation and discussion of the analysis study cases and results are included in Appendix F.

Geometry and Boundary Conditions

The geometry of the embankment and soil stratification was developed from the current topography maps and geotechnical investigation of the project. The reservoir water levels in the models are summarized in Table 15. The potential seepage boundary condition with zero flux is applied to the downstream face of the embankment as well as the ground surface downstream of the toe of the dam in all models.

Material Properties

The material properties selected for the different material types are discussed in Section 7.0 and presented in Tables 2 through 5 in Appendix F. The material types are identified by color on the model cross-sections on Figures 1 through 6 in Appendix F.

The permeability curves of the partially saturated materials such as embankment and foundation soils were estimated using the Fredlund and Xing method in the SEEP/W manual (GeoSlope, 2010) up to a maximum matrix suction of 10,000 psf. The residual water content of the materials was also estimated using the method indicated in the SEEP/W 2007 manual.

SEEP/W Results

The output plots of the analysis are presented in Appendix F on Figures 7 through 14 for BC No. 1 and Figures 15 and 16 for BC No. 2. Analysis results indicate that the location of the phreatic surface would be similar for the lower and upper bound permeability values used in the analyses. The results also indicate that a functioning toe drain for the BC No. 1 dam would have a significant impact on the location of the phreatic surface (see Figures 9, 10, 13, and 14 in Appendix F). The pore water pressure values from the SEEP/W analyses were transferred to SLOPE/W models for estimating the slope stability FOS.

10.2 Embankment Stability Analysis Results

Slope stability analyses were performed using the GeoStudio 2007 version 7.17 computer program to estimate the FOS for static and post-earthquake loading conditions for BC No. 1 and BC No. 2. Static and post-earthquake shear strength values presented in Section 7.0 and discussed further in Section 9.0 above

were utilized. The Spencer's method of slices was used to perform the analyses since it satisfied both force and moment equilibrium of each slice. The geometry of the stability analysis models were the same as the geometry of the models used in the seepage analyses.

BC No. 1

The results of the stability analysis are summarized in Table 16. The minimum FOS values, estimated for the static loading conditions at BC No. 1, exceed 1.5 for both Study Case 1 (without toe drain) and Study Case 2 (with toe drain). An example of the results for the downstream slope at the BC1-CPT-4 cross section for Study Case 2 and drained strength parameters are shown on Figure 31.

Post-earthquake analysis results using reduced shear strength values are also summarized in Table 13. Undrained Strength Values for Post-Earthquake Slope Stability Analyses used in 1974 analyses by CH2MHill, Dam BC No. 2. As can be seen, the minimum post-earthquake FOS values are significantly lower than the static values. The greatest reduction in the estimated minimum FOS occurs using the cross-section characteristics and reduced shear strength values for BC1-CPT-3. The most critical potential failure surface corresponding to the estimated minimum FOS of 1.08 is shown on Figure 31 and extends into the liquefiable, sand-like soil foundation soils. The failure surface extends to a daylight location below the reservoir water surface elevation suggesting that an overtopping failure mode could develop if deformations become large enough. The minimum post-earthquake FOS results using the cross-section and reduced strength values for BC1-CPT-4 are 1.44. The critical potential failure surface corresponding to this minimum FOS value is shown on Figure 33. These results are also highlighted yellow. In both cases, the results suggest that additional evaluations of the downstream slope of BC No. 1 should be performed to further refine the cross-section properties and estimate deformations of the structure using more advanced numerical modeling methods to determine the potential for an overtopping or a cracking/seepage related failure mode to develop during a large earthquake event. Based on our experience, HDR believes that the ground motions associated with a CSZ M 9.0 megathrust event will be the critical safety and design event for this dam.

One of the significant characteristics of subduction zone earthquakes around the world is the occurrence of significant after shock events a relatively short time after the primary event occurs. The strength reduction to the clay-like soils associated with the BC1-CPT-4 cross section would likely occur during the initial and primary earthquake event. Pore water pressures that would develop in the high plasticity clayey silt materials in the embankment and foundation of the dam would not likely dissipate for several weeks allowing a corresponding return to a higher shear strength and minimum FOS conditions. Hence, any subsequent earthquake response would begin at the condition of reduced shear strength and additional significant deformations may be induced to the structure.

To make an initial assessment of this concern, a pseudostatic seismic analysis was performed to estimate the yield acceleration (i.e., FOS=1.0) for each case using the reduced shear strength parameters. The results for the downstream slope using strength values for BC1-CPT-3 are shown on the upper portion of Figure 34. The estimated yield acceleration for BC1-CPT-3 is about 0.006g (upper graph). This low yield acceleration (the acceleration to cause additional structure deformation) is expected because the post-earthquake minimum FOS was only 1.06. For BC1-CPT-4 conditions, (lower graph), the yield acceleration is only 0.04g, even though the post-earthquake minimum FOS was 1.49. These results suggest that aftershocks will be a significant consideration in the assessment of the overall safety of BC No. 1 and design of any remediation treatments.

BC No. 2

The results of the stability analysis are summarized in Table 17. The minimum FOS value of 1.83 estimated for the static loading condition of the downstream slope of BC No. 2 also exceed 1.5. The critical potential failure surface associated with this minimum FOS is shown on Figure 35.

Post-earthquake analysis results using reduced shear strength values are also summarized in Table 17. As can be seen, the minimum post-earthquake FOS value of 0.4 is less than 1.0 suggesting a significant potential for a stability failure of the structure during a large earthquake. The location of the critical failure surface associated with this minimum FOS value is shown on Figure 36. The failure surface daylights substantially below the reservoir and sediment levels strongly suggest the corresponding development of an overtopping failure mode releasing the full contents of the reservoir at the time of the earthquake. The minimum FOS value for the downstream slope results are highlighted red in Table 17. It should be noted that the minimum FOS value for the upstream slope is well above 2.0 suggesting that only the safety of the downstream slope requires further evaluation and corrective action. Similar to BC No. 1, based on our experience, HDR believes that the ground motions associated with a CSZ M 9.0 megathrust event will be the critical safety and design event for this dam.

11.0 CONCLUSIONS AND RECOMMENDATIONS

11.1 Conclusions

BC No. 1

The minimum FOS identified for BC No 1 (lower) indicates that this structure meets acceptable stability criteria and is stable under static loading conditions using the estimated static strength of the soils.

The BC No. 1 clay-like embankment soils are not well compacted, and the relatively loose sand-like and clay-like foundation soils extend up to 60 feet below the embankment. Based on the limited geotechnical explorations that were performed for this preliminary seismic evaluation, liquefaction of the relatively loose sand-like soils would result in a considerable loss of soil shear strength during a large earthquake event. The strength of the clay-like embankment and foundation soils would also be reduced in a seismic event. Simplified post-earthquake stability analysis results using the estimated reduced shear strength of these materials (that would occur during an earthquake) indicated that BC No. 1 could be susceptible to damage due to a large seismic event originating on either the Yaquina fault or CS Z. The dam may be subject to further and significant damage associated with aftershocks. Either fault system can generate large earthquakes, but the CSZ is of greater concern because of the relatively long duration of strong shaking from subduction-type earthquakes.

Field studies completed as part of this evaluation identified that the discharge end of the drainage blanket under the downstream embankment slope is not exposed as originally designed and constructed. This drain appears to be covered by up to 8 feet of clay-like soil fill (Figure 3). While the soils covering the drain discharge may slightly enhance the stability of the downstream slope, the drain is likely not functioning resulting in an increase in the water pressures in the dam and foundation materials beneath the downstream slope. The available records do not indicate when and why this fill was placed. Restoration of the drainage blanket function should be considered as part of future evaluation and remediation designs.

BC No. 2

The minimum FOS value identified for BC No. 2 (upper) indicates that this structure meets acceptable stability criteria and is stable under static loading conditions using estimated static strength of the soils.

As simplified analysis results indicated, however, the downstream slope of BC No. 2 is susceptible to significant damage and would likely experience a stability failure due to a seismic event originating on either the Yaquina fault or CSZ. Either fault system can generate large earthquakes, but the CSZ is of greater concern because of the relatively long duration of strong shaking from subduction type earthquakes. The critical potential failure surface identified in these evaluations suggest that an overtopping breach of the dam would occur releasing the full contents of the reservoir.

The BC No. 2 clay-like embankment soils are generally well compacted; however, loss in strength of some of the clay-like embankment soils, particularly in the lower portions of the embankment and cutoff trench could still occur because of the intensity of ground shaking that is possible. Based on the available design and construction records, it appears that most of the alluvial soils were removed for construction of the cutoff trench; however, outside of the relatively narrow cutoff trench the embankment dam was constructed on the alluvial foundation soils that also appear to have the potential for significant strength loss during earthquake loading. One boring drilled near the downstream toe of the embankment dam (BC2-B-3) also revealed a relatively loose layer of potentially liquefiable sand-like soil. Liquefaction of this relatively loose layer of sand-like soil would also result in a considerable loss of soil strength.

11.2 Recommendations

The preliminary seismic evaluation of the BC No. 1 and BC No. 2 dams presented in this report has indicated significant safety concerns with each dam. It is noted however, that these evaluations were based on limited site characterization information and a simplified analyses procedure. Safety concerns as well as any remediation design are sensitive to the characterization of the embankment dam and foundation soils. The differentiation between the sand-like liquefiable soils and the clay-like soils and the corresponding post-earthquake strength of materials that may be susceptible to liquefaction or cyclic softening is a critical consideration and is dependent on the density and PI of the soils. The loss of strength of sand-like soils due to liquefaction during seismic loading is the more acute consideration at the site.

Based on the results of this evaluation and experience on similar projects including the nearby Scoggins Dam evaluations underway by the U.S. Bureau of Reclamation, HDR recommends that an additional phase of site characterization studies including additional sampling and testing of the embankment and foundation soils along with correlation of soil properties to existing and additional CPT soundings be completed. Further, we recommend that more advanced numerical modeling of the dams be performed to support the safety assessment and for development of remediation concepts. Laboratory testing of soil samples is the only means to reliably classify the soil as either sand-like or clay-like and to support the development of estimates of peak and reduced undrained shear strength.

Additional Field Exploration and Laboratory Testing

To properly characterize the soils, HDR recommends drilling three additional borings at BC No. 1 and four additional borings at BC No. 2. Each boring would be drilled at least 10 feet into the decomposed/weathered siltstone. Since the foundation soils are highly variable, soil samples spaced on 2.5-foot intervals is required. At each boring location, a boring will be drilled utilizing the SPT sampler to obtain disturbed samples to determine the soil PI. Based on the field classification of the soils, a companion boring will be drilled next to the SPT boring to obtain undisturbed samples with a hydraulic fixed-piston sampler. This will provide the highest quality undisturbed samples for laboratory testing. Such a program will target samples from the optimum depth and will result in the minimum number of required undisturbed samples and laboratory testing. Laboratory testing of the undisturbed samples should include consolidation, static triaxial, and static and cyclic direct simple shear.

Dam Repair Alternatives Analysis

The seismic evaluation of each dam would be revised based on the results of the additional boring and laboratory test data. If these results indicate that the dams are still vulnerable to damage during a seismic event, repair alternatives should be developed. Based on the workshop held at HDR's Portland office on August 2, 2012, it is understood that the City of Newport may not want to repair BC No. 1 even if the analysis indicates the dam could fail during a seismic event. HDR recommends that alternatives be developed for BC No. 1 that include a conceptual design and cost estimate to allow the City to then decide if the cost to repair BC No.1 is prohibitive and if storage from the BC No. 1 reservoir should be moved to BC No. 2 with a corresponding enlargement of that dam and reservoir.

Repair of BC No. 1 Drainage Blanket

As previously noted, restoration of the downstream embankment drainage blanket function should be considered as part of future evaluation and remediation designs.

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PLANNING STAFF MEMORANDUM
FILE No. 3-CP-12

I. Applicant: City of Newport. (Initiated pursuant to authorization of the Newport Planning Commission).

II. Request: Revisions to the Urbanization and Public Facilities elements of the Newport Comprehensive Plan to update standards against which an Urban Growth Boundary amendment is evaluated (i.e. implementation of Goal 14, effective 2006), establish that it is city policy to acquire lands and protect water quality within its municipal watershed, acknowledge structural deficiencies in the city municipal water reservoirs, and outline steps the city will take to resolve the deficiencies.

III. Planning Commission Review and Recommendation: The Planning Commission will review the proposed amendments and provide a recommendation to the City Council. At a later date, the City Council will hold an additional public hearing prior to any decision on the amendments.

IV. Findings Required: The Newport Comprehensive Plan Section entitled "Administration of the Plan" (p. 287-288) requires findings regarding the following for the proposed amendments:

A. Data, Text, Inventories or Graphics: (1) New or updated information.

B. Conclusions: (1) Change or addition to the data, text, inventories, or graphics which significantly affects a conclusion that is drawn for that information.

C. Goals and Policies: (1) A significant change in one or more conclusion; or (2) a public need for the change; or (3) a significant change in community attitudes or priorities; or (4) a demonstrated conflict with another plan goal or policy that has a higher priority; or (5) a change in a statute or statewide agency plan; or (6) applicable statewide planning goals.

D. Implementation Strategies: (1) a change in one or more goal or policy; or (2) a new or better strategy that will result in better accomplishment of the goal or policy; or (3) a demonstrated ineffectiveness of the existing implementation strategy; or (4) a change in the statute or state agency plan; or (5) a fiscal reason that prohibits implementation of the strategy.

These findings are addressed in the proposed ordinance, attached to this report.

V. Planning Staff Memorandum Attachments:

Attachment "A"	Draft of the proposed Ordinance, with exhibits
Attachment "B"	Dam Assessment PowerPoint, prepared by HDR, dated August 2012
Attachment "C"	DEQ/OHD Source Water Assessment Summary (PWS # 4100566)
Attachment "D"	Statewide Planning Goal 14, Urbanization, effective April 2006
Attachment "E"	1993 Park System Master Plan Reservoir Regional Park Concept
Attachment "F"	Notice of Public Hearing
Attachment "G"	Markup copy of revisions to Public Facilities and Urbanization Elements of the Newport Comprehensive Plan

VI. Notification: Notification for the proposed amendments included notification to the Department of Land Conservation & Development (DLCD) in accordance with the DLCD requirements on December 14, 2012. Notice of the Planning Commission hearing was published in the Newport News-Times on January 16, 2013 (Attachment "F").

VII. Comments: As of January 23, 2013, no written comments have been submitted on the proposed amendments. Staff met with representatives from DLCD on January 7, 2013 to review the proposed changes. They advised that the agency didn't have any issues with the revisions at that time.

VIII. Discussion of Request: Revisions to the Public Facilities element of the Newport Comprehensive Plan were initiated by the Planning Commission in response to a Dam Assessment, performed by HDR consultants, which identifies structural deficiencies in the City of Newport's Big Creek #1 and Big Creek #2 domestic water storage reservoirs (Attachment B). This new information is not addressed in the City's Water System Master Plan, which was last amended in 2008. While the full extent of the deficiencies is not yet known, it is evident from the analysis that the City will need to reconstruct one or both of its reservoirs. Proposed policies describe how the City should respond to this threat to its domestic water supply, including strategies for completing necessary engineering studies to ascertain the full scope of the problem, financing future construction and land acquisition, and protecting water quality consistent with a source water assessment performed by the Oregon Dept. of Environmental Quality/Oregon Health Department (Attachment C). A policy referencing an outdated Public Facility Plan from 1990 is being deleted.

A second set of amendments are proposed to the Urbanization element of the Newport Comprehensive Plan. This section of the Comprehensive Plan sets out the process and criteria for amending the Newport Urban Growth Boundary (UGB). Changes to a municipal UGB must comply with Statewide Planning Goal 14 (Attachment D). That statewide planning goal was updated in April of 2006. The City Council has expressed an interest in expanding the Newport UGB to include the reservoirs under urban "public" zoning that would make it easier for the City to modify its water infrastructure in response to the reservoir structural issues and to construct a future regional park called for in the 1993 Park System Master Plan (Attachment E). To efficiently accomplish this objective, the Commission felt it prudent for the City to update its standards for evaluating UGB amendments to comport with current state law before considering an expansion proposal involving the reservoirs. Proposed revisions include changes to the text of the section to reflect that the City's official population forecast has been updated to 2031, clarification regarding areas that have been studied for potential future inclusion into the Newport UGB, and amendments to the required findings section that set out the needs analysis required under the current (2006) version of Goal 14.

A markup copy of the specific changes is enclosed (Attachment G). A placeholder for a detailed geotechnical examination of the reservoirs is included in the draft ordinance. That report, which is in draft form, builds upon the analysis contained in the HDR PowerPoint presentation.

IX. Conclusion and Recommendation: The Planning Commission should review the proposed amendments and make a recommendation to the City Council. As this is a legislative process, the Commission may recommend changes to the amendments if the Commission chooses to do so. The City Council may also make changes to the proposal prior to adoption of a final decision.

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City of Newport

January 23, 2013